APPENDIX 5-16

WAVE RUN-UP TECHNICAL MEMORANDUM 4
FROM WORK ORDER 3

BLACK & VEATCH

TECHNICAL MEMORANDUM

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Evaluation of Wave Run-up and Internal Breakwaters

To: Distribution

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1. OBJECTIVE

The overall objectives of the Wave Run-up Model are as follows:

- To determine the amount of freeboard required to prevent over-topping of the reservoir embankment during high wind and rain conditions
- To determine the effectiveness of internal breakwaters in decreasing wave run-up

This memorandum summarizes the final tasks completed in developing the Wave Run-Up model and includes the results of the freeboard analysis and the internal structures evaluation using the selected model and United States Army corps of Engineers (USACE) procedures. The USACE procedures relevant to this analysis include the following:

- Determine fetch distances for wave generation and for wind set-up
- Determine wave characteristics for each design case
- Determine the wind set-up
- Calculate the wave run-up
- Calculate wave transmission through the internal breakwaters and resultant wave run-up on the embankments

2. DESIGN CONDITIONS

Design conditions to be modeled for the EAA Reservoir A-1 were identified in Work Order 3, Technical Memorandum 1, Model Conditions and Design Conditions for Wave Run-Up Model and were further defined in Technical Memorandum 3, Wind and Rainfall Analysis. Three design conditions were recommended in Technical Memorandum 3. These design conditions were based upon historical data that was obtained from NOAA hurricane reports and NOAA weather stations.

The three recommended design conditions included:

- Condition 1: A category one hurricane with normal rainfall associated with a hurricane of that magnitude. The recommended wind and rainfall amounts for the first condition are wind speed of 85 mph with a rainfall of 12 inches
- Condition 2: A hurricane event in combination with the Probable Maximum Precipitation (PMP). The recommended wind speed for the second condition is 80 mph in conjunction with the PMP
- Condition 3: A hurricane with very strong winds and rainfall normally associated with that type of hurricane. The recommended wind and rainfall amounts for the third condition are wind speed of 132 mph with a rainfall of 7 inches

At the same time, the South Florida Water Management District (District) and the USACE held a design workshop to pick appropriate design conditions for the EAA Reservoir A-1. The design conditions selected by the District and USACE and evaluated in this Work Order included:

- 200 mph wind with no rainfall
- 105 mph wind with the PMP

These conditions were evaluated with and without internal breakwaters that could reduce wave run-up on the embankments.

2.1 Design Condition 1: 200 mph Wind Event

The first design condition evaluated was a 200 mph wind with no rainfall. This is a significantly higher wind speed than was recommended by B&V. To put the 200 mph wind event in perspective, a discussion of a wind event of that magnitude follows.

Category 5 hurricanes are those with measured or estimated wind speeds greater than 155 mph. Typhoon Nancy on 12 September 1961 is listed as having an estimated maximum sustained wind of 213 mph, but it is now recognized that hurricane wind estimates from the 1940s to 1960s were too high (NOAA, 2005a). In the North Atlantic, both Hurricane Camille (1969) and Hurricane Allen (1980) had winds estimated at 190 mph (NOAA, 2005a). Only three category 5 hurricanes have had landfall on the continental U.S. (NOAA, 2005b): the “Labor Day” hurricane in 1935 (landfall on the Florida Keys), Camille in 1969 (Landfall on Mississippi and Southeast Louisiana), and Andrew in 1992 (landfall on Southeast Florida). Of these, only Camille had an estimated wind speed at landfall of 190 mph (NOAA, 1991). Hurricane Gilbert was also a category 5 hurricane when it made landfall on the Yucatan Peninsula, with wind speeds between 130 and 145 mph (NOAA, 1991).

In 2004, the tracks of Hurricane Charley, Frances, and Jeanne intersected over Polk County, Florida, about 100 miles northeast of the reservoir site (Bossak, 2004). Hurricane Charley made landfall near Cayo Costa on the West Coast of Florida as a category 4 hurricane with maximum winds of 150 mph. Maximum sustained winds observed at Winter Haven and Orlando were between 50 and 80 mph, with gusts to 105 mph (NOAA, 2004a). Hurricane Frances made landfall over the Southern end of Hutchison Island on the East Coast of Florida as a category 2 hurricane with maximum winds of 105 mph. The 15-minute average sustained wind was 60 mph at West Lake Okeechobee, with gusts to 90 mph (NOAA, 2004b). Hurricane Jeanne also made landfall over the Southern end of Hutchison Island as a category 3 hurricane, with maximum winds of 120 mph, but it was uncertain if these strongest winds reached the coast. Maximum sustained winds observed at Fort Lauderdale and Orlando were between 40 and 60 mph, with gusts to 75 mph (NOAA, 2005c).

In defining a wind to be used in the modeling, duration and height of observation must be identified in addition to wind speed. The strongest wind speeds reported in the hurricane reports are typically 1 minute averages. Frequently during hurricanes standard reporting stations experience equipment malfunctions due to the storm itself. Therefore, wind speed is often recorded using remote sensing equipment (radar) or instruments dropped from airplanes through the atmosphere. With either of those techniques, reported wind speeds will be observed at much higher elevations that the 10 m standard used at meteorological stations. Because of boundary layer effects, winds speeds at 10 m can be significantly lower that those measured at higher elevations. If winds observed at higher elevations are used in the modeling to represent wind measured at 10 m, the effects of wind will be overestimated.

Personnel at NOAA have developed a model to forecast the maximum wind of landfalling hurricanes or to predict the maximum inland penetration of hurricane force winds (NOAA, 2004c). The Inland Wind Model and Maximum Envelope of Winds (MEOW) indicates that a category 5 hurricane making landfall on the east coast of Florida would have winds of about 126 mph at Lake Okeechobee. It appears that the 200 mph in Design Condition 1 is conservative.

2.2 Design Condition 2: PMP with 105 mph Wind

The second design condition includes a 105 mph wind with the PMP. The 105 mph wind is a mid-range category 2 hurricane. The 72hr-PMP will be determined under Work Order 5. However, an initial estimate of the PMP was made for the wave run-up modeling. The estimated PMP rainfall followed HMR 51 and HMR 52 guidelines but only included Step A of the procedure. The estimated 72-hr PMP rainfall at the reservoir site was 54 inches of rain, or 4.5 ft. If a category 2 hurricane in conjunction with the PMP were to affect the reservoir site, the hurricane force winds would likely subside before all of the rainfall occurred. However, as a conservative assumption, this condition was modeled with all of the rainfall occurring prior to the maximum wind event. For this design condition, reservoir depths of 12 and 15 feet were modeled as depths of 16.5 and 19.5 feet. The change in reservoir depth affects wave growth and wind set-up.

2.3 Other Variables

In addition to the two design conditions discussed above, several other variables were modified to simulate a range of design conditions. These variables included fetch, depth, slope and type of surface on the embankment.

Evaluation of Wave Run-up and Internal Breakwaters

Prior to considering the effects of internal breakwaters eight cases for wind, rainfall, water depth and embankment slope were considered:

- Case 1: 200 mph wind, no rainfall, 12 ft water depth, 3 H to 1V slope
- Case 2: 200 mph wind, no rainfall, 12 ft water depth, 1 H to 2 V slope
- Case 3: 200 mph wind, no rainfall, 15 ft water depth, 3 H to 1 V slope
- Case 4: 200 mph wind, no rainfall, 15 ft water depth, 1 H to 2 V slope
- Case 5: 105 mph wind, PMP, 12 ft water depth, 3 H to 1V slope
- Case 6: 105 mph wind, PMP, 12 ft water depth, 1 H to 2 V slope
- Case 7: 105 mph wind, PMP, 15 ft water depth, 3 H to 1 V slope
- Case 8: 105 mph wind, PMP, 15 ft water depth, 1 H to 2 V slope

For each of these cases two effective fetch distances and a smooth and rough surface were modeled, resulting in a total of 32 cases.

3. MODEL CONFIGURATION

The ACES (Automated Coastal Engineering System) program was used to calculate wave growth, wave run-up and wave transmission through internal breakwaters. One of the important assumptions made in the model is that the water depth is constant. The calculations were completed using the wave prediction, wave theory, wave run-up and wave transmission modules of the program. The ACES model does not calculate wind set-up and this was calculated separately using the Sibul model.

3.1 Wave Prediction

The wave prediction section of the model computes wave growth. Wave growth is a function of the speed and duration of winds, fetch distance, and water depth. It was assumed that winds were observed at a height of 10 m. Wind speeds of 105 and 200 mph were modeled and it was assumed that peak winds had a duration of 0.1 hours. Water depths of 12 and 15 ft were modeled. Effective fetches were calculated and modeled as shown on Figure 1. The outputs produced by the model include the effective fetch, adjusted wind speed, mean wave direction, wave height, and wave period.

3.2 Wave Theory

The energy associated with the waves is calculated in the wave theory section. The required inputs include wave height, wave period, breaking criteria, and water depth. The outputs produced by the model include wave length, energy flux, and group velocity.

3.3 Wind Set-Up

Wind set-up can be an important factor in determining freeboard requirements. Wind set-up occurs when wind blows in a relatively constant direction over the water surface. Shear stresses between the wind and water exert a drag on the water and pushes the water in the direction of the wind. When the water encounters a barrier such as a shoreline or embankment it piles up resulting in deeper water at the shoreline. Because the mass of water in the reservoir will be conserved, a decrease in water depth will occur at the leeward side of the reservoir to offset the wind set-up. However, the slope of the water surface is curved, not linear so the decrease in depth at the leeward side of the reservoir will not equal the increase in depth at the windward side of the reservoir.

Wind set-up will increase until there is a balance between the shear stresses on the water surface and a gravity induced return flow along the reservoir bottom. Wind set-up is a function of wind speed, fetch, and water depth. Wind set-up increases with wind speed and fetch but decreases with increasing water depth.

Wind set-up is not included in the ACES model. The Sibul model was used to calculate wind set-up (USACE, 2004) and the results were added to the wave run-up calculations. The Sibul model is an empirical relationship based on the numerical model developed by Brater et al. (1996). The empirical relationship used laboratory and field data including data collected at Lake Okeechobee. The design conditions evaluated in this Technical Memorandum were not represented in the data used to develop the empirical relationship. Therefore, there is uncertainty in using the model for very high wind speeds. The following excerpt from USACE (2004) discusses the reduction in drag coefficient observed by Powell et al. (2003).

> *Experiments with high wind speeds over saltwater show an unexpectedly [sic]drop in the drag coefficient as speeds increase from approximately 90 mph to 114 mph (Powell). A possible explanation as suggested was that as wind speeds increase above hurricane force, the surface becomes layered in foam that may impede the transfer of momentum from the wind, essentially creating a "slip" surface. The reduced wind drag coefficient as observed appears to decrease to a range from 2.5 x 10^{-6} to 2.0 x 10^{-6} with the lowest expectation around 1.5 x 10^{-6}. . . . Remembering that these experiments were on saltwater, it is uncertain at this time whether or not these same observations may be expected on freshwater reservoirs.*

3.4 Wave Run-Up

The wave run-up section of the model calculates the run-up that occurs when waves encounter a shoreline or embankment. The required inputs include wave type, breaking criteria, wave height, wave period, structure slope, structure height, slope type, and roughness coefficient. The cases modeled included both steep and smooth slopes with both smooth and rough surfaces. Roughness coefficients consistent with rip rap were used for the cases with rough surfaces.

The output calculated by the model includes wave run-up, deepwater wave height, and wave steepness. Figure 2 indicates how the waver run-up parameters are defined. Wave run-up (R) is measured from the still water level as opposed to wave height (H) which is measured from trough to crest.

3.5 Wave Transmission

The wave transmission section of the ACES model was used to determine the effects that internal breakwaters would have on wave growth, transmission of wave energy through internal breakwaters and on final wave run-up on the embankments. Required inputs include information on the breakwater, water depth, and incident wave height and period. Required inputs for the breakwaters include physical dimensions, median diameter of material, and porosity.

For this analysis, two configurations of internal breakwaters were evaluated. One configuration, shown in Figure 3, is a peripheral wall approximately ½ mile inside of the embankment. This concept also includes several short breakwaters, perpendicular to the peripheral wall, to minimize the fetch along the channels between the embankments and breakwaters. Figure 4 shows a different potential configuration of internal breakwaters. This concept includes a circle breakwater in the middle of the reservoir with several spokes radiating toward the embankments. The peripheral wall minimizes the fetch distance between the breakwater and the embankments. However, the peripheral wall is longer than the circle and would be more expensive to construct.

4. MODEL CALIBRATION, VERIFICATION and RELIABILITY

Conventional methods for performing calibration and verification of the ACES wave model are not available since data for the design conditions do not exist. However, the ACES model is based upon empirical measurements from other reservoirs and scientific investigations. Therefore, it is based upon real data and is appropriate for normal applications and uses of it modules. However, the first design condition for the EAA Reservoir A-1, the 200 mph wind event, is likely outside of the empirical relationships developed within the ACES model.

The wave prediction module of ACES is “based upon the fetch-limited deepwater formulas, but modified to include the effects of bottom friction and percolation” (Bretschneider and Reid, 1954). These relationships have not been verified. Depth is considered to be a constant in the equation in this module. Although the wind set-up in the lake will produce a slight slope ($3x10^{-4}$) in the water surface, an appropriate assumption for this analysis was to set the depth input as the average depth across the reservoir.

The wave theory module is based upon calculations derived by Airy (1845) and considers the following set of assumptions:

- Waves are two dimensional (2D) in the x-z plane
- Waves propagate in a permanent form over a smooth horizontal bed of constant depth in the positive x-direction
- There is no underlying current
- Fluid is incompressible, has no surface tension, and has no viscosity
- Flow is irrotational
- Coriolis effect is neglected

Evaluation of Wave Run-up and Internal Breakwaters

The assumptions used in the wave prediction module are appropriate for the EAA Reservoir A-1. The EAA Reservoir A-1 will be a flat bottomed reservoir that is an open body (i.e. not dendritic) in nature. The reservoir footprint is small in comparison to the Coriolis effect. The underlying current, which may occur due to wave-setup will be low in velocity. During the wave setup, return current flow will be moving away from the reservoir's edge as the hydrostatic forces try and re-level the reservoir. This return flow would be largest at the reservoir's edge. It was assumed any current would be negligible.

Wave transmission and wave run-up modules were derived from physical model studies originally conducted for specific structures and wave climates (Leenknecht, 1992). General assumptions for the wave run-up on an impermeable embankment are:

- Waves are monochromatic, normally incident to the structure, and unbroken in the vicinity of the structure toe
- Waves are specified at the structure location
- All structure types are considered to be impermeable
- For sloped structures the crest of the structure must be above the still-water level.
- For vertical and composite structures, partial and complete submersion for the structure is considered
- Run-up estimates on sloped structures require the assumption of infinite structure height and a simple plane slope
- The expressions for the transmission by overtopping use the actual finite structure height

The wave transmission through a permeable structure (i.e. wave break) is estimated by relationships developed by Madsen and White (1976) and Seelig (1980). General assumptions for the wave transmission through a porous embankment are:

- Incident waves are periodic, relatively long, and normally incident
- Fluid motion is adequately described by the linearized governing equations
- The model can be used only for crests above the still-water line
- The model can be used for unbroken waves

The assumptions for wave run-up and wave transmission modules chosen to model the EAA Reservoir A-1 are appropriate. The design of the A-1 embankment will be both impermeable and taller than the combination of wave run-up, wave height, and wind set-up for the chosen design conditions. The design of the embankment will have no overtopping. The wave transmission module gives the best predictions for shallow-water waves which fits the water depths of the EAA Reservoir A-1. Both modules are based upon physical model studies and their reliability is well documented.

5. RESULTS

5.1 Wave Characteristics

The wave growth and wave theory sections of the ACES model were used to identify the wave characteristics that could occur under the design conditions. The wave characteristics of wave height, period and length are summarized in Table 1 for all cases without internal structures. The wave length, period and height all increase with increasing fetch, depth and wind speed.

The fetch lengths are shown on Figure 1. Two fetch lengths, the longest and shortest, are included in Table 1. Figure 5 shows how wave height increases with increasing fetch. Two conditions are presented:

- 200 mph wind with 12 ft water depth and effective fetches ranging from 0.1 to 8.5 miles
- 105 mph wind with 16.5 ft water depth (12 feet plus 4.5 feet of rainfall from the PMP) and effective fetches ranging from 0.1 to 8.5 miles.

Figure 5 indicates that for both conditions, there is an initial sharp rise in wave height with increasing fetch. However, beyond a distance of about three (3) miles, both curves become relatively flat indicating that an increase in fetch has a lesser effect on wave height. The relationship of wave height to fetch is an important factor in the placement of internal breakwaters. Breakwaters placed in the middle of the reservoir could still produce some relatively long fetches and allow the regeneration of waves between the breakwaters and embankments.

5.2 Wind Set-up

Wind set-up calculations were made for each of the cases evaluated. Results of these calculations are presented in Table 2. For the first case presented, a wind speed of 200 mph, fetch of 8.5 miles and depth of 12 ft, the predicted wind set-up was 11.9 ft. Therefore the water depth at the embankment would be double the original water depth.

However, wind set-up is a function of water depth and it seems reasonable to assume that as water depths increase along the embankments, the bottom return flow would increase and limit the wind set-up. Therefore, an adjustment was made to the water depth in the calculations. The water depth was increased to an amount equal to 25 percent of the calculated wind set-up. If the water piled up along the embankment was described as a triangle of water above the normal water level, the increase of 25 percent of the water depth would be equivalent to the average depth in the triangle. Adjusted wind set-up values were calculated using the adjusted water depths. For the cases that included the 200 mph winds, this adjustment reduces the wind set-up by 7 to 18 percent. For the cases with 105 mph winds there is little difference in the original and adjusted wind set-up values.

5.3 Wave Run-up

The wave-run-up module of the ACES model was used to estimate wave run-up for each of the 32 cases evaluated. The results for cases without internal breakwaters are presented in Tables 3 through 10. Each table lists the wind speed, depth of reservoir, embankment slope, surface roughness, fetch, wave height, rainfall amount, effective depth, wind set-up, wave run-up, minimum freeboard, and minimum embankment height.

Evaluation of Wave Run-up and Internal Breakwaters

The effective depth is the reservoir design depth plus rainfall. The effective depth was used to generate wave characteristics including wave height. The freeboard requirement is the sum of rainfall, wind set-up and wave run-up. The embankment height is the sum of effective water depth, wind set-up and wave run-up. Also, as previously noted, the wave run-up is measured from the still water level while the wave height is measured from trough to crest.

Wave run-up increases with an increase in incident wave height and with the slope of embankment. The wave run-up is not directly related to water depth and therefore is not related to wind set-up. Wave run-up is indirectly related to water depth because depth affects the incident wave height. On the flatter slope, the run-up was greater for a smooth surface; on the steeper slope, run-up was greater for the rough surface.

5.4 Effects of Internal Structures

For this analysis, two configurations of internal breakwaters were evaluated; a peripheral wall approximately ½ mile inside of the embankment, and a circle breakwater in the middle of the reservoir with several spokes radiating toward the embankments. For the peripheral wall, a wind direction in an approximate northwest to southeast direction though the reservoir would cover the longest fetch and was selected as the wind direction to be modeled. This wind direction is shown on Figure 6. For the circle breakwater, two wind directions were selected. The northwest to southeast wind direction would have the longest overall fetch and was therefore selected for modeling. Another wind alignment in a more north to south direction was also selected because it produced different intermediate fetch lengths. These two wind directions are shown on Figure 7.

The design of internal breakwaters, if any, will be completed at a later stage in the project. However, assumptions on size and materials were made for modeling purposes. It was assumed that the breakwaters would extend above the water surface; so for a depth of 12 ft and a PMP of 4.5 ft, the minimum height of the breakwater would be 16.5 ft. A breakwater height of 20 ft was used in the modeling. It was also assumed that the side slopes would be 2H to 1V, the mean diameter of breakwater material would be 1 ft and the porosity would be 30 percent. It was further assumed that the top width of the breakwaters would be 8 ft, resulting in a bottom width of 88 ft. It is recognized that any of these assumptions can change as more information is developed on the available materials and costs of construction. Additional wave run-up calculations will be conducted in Work Order 5 to reflect new information and design concepts.

For each of the configurations and wind directions evaluated, wind and waves would encounter two breakwaters. Therefore, a step-wise approach was taken to develop the wave run-up on the reservoir embankment.

1. Starting at the leeward end of the reservoir wave heights were calculated considering a depth of 12 ft and the effective fetch between the embankment and the first breakwater.
2. The wave transmission module of the ACES model was then used to calculate the characteristics of the transmitted wave, primary wave height and period.

Evaluation of Wave Run-up and Internal Breakwaters

3. Wave growth between the first and second breakwater was then calculated based on wind speed, water depth, and fetch. The formulas used to calculate wave growth assume that the water surface is originally flat. However, in this case a transmitted wave is present. To account for the presence of the transmitted wave an adjustment was made to the fetch. Essentially, the fetch required to generate the transmitted wave was added to the fetch between the structures to determine the wave height of the wave at the second breakwater.
4. Using the new wave characteristics, the wave transmission through the second breakwater was then calculated.
5. Wave growth between the second breakwater and the embankment was calculated again based on wind speed, water depth and fetch. An adjustment was made to the fetch distance to account for the transmitted wave as described in step 3.
6. Finally, the wave run-up was calculated based on the final wave characteristics calculated in step 5.

One of the factors that affects the wave transmission through the breakwaters is the water depth at the structure. As the water depth increases, and the amount of the structure that is above the water line decreases, the amount of wave energy transmitted past the structure will increase. Therefore, the effects of wind set-up on the internal breakwaters should be considered in the analysis because if wind set-up is causing water to pile up at the internal breakwaters, wave transmission will be higher and the breakwaters will not be as effective at reducing wave run-up. However, because the breakwaters will not be impermeable, it is more difficult to calculate wind set-up on the structures.

The Sibul Model was used to calculate the wind set-up on the internal breakwaters. The calculation was initially made as if the breakwater were impermeable structures such as the embankments. It was then estimated that the wind set-up on the permeable breakwater would be 75 percent of the amount calculated. The depth of water due to the wind set-up was added to the original water depth to obtain the depth of water at the breakwater. This new water depth was then used to analyze the transmission through the breakwater. The assumption of 75 percent is an estimate and the actual amount of wind set-up could be much lower because of the porosity of the breakwater. However, this was considered a conservative yet reasonable assumption because a lower value for wind set-up would yield a more effective breakwater. Assuming a lower value for wind set-up on the internal breakwaters would result in higher wind set-up at the embankment but smaller wave run-up. As the design concepts for the internal breakwaters become more defined it would be useful to run a sensitivity analysis for the wind set-up assumption to determine the overall effect of this assumption on freeboard requirements.

The effects of the peripheral wall on wave run-up are summarized in Table 11. All cases shown are for a 12 ft deep reservoir. For this breakwater configuration, the results of the wave transmission analysis indicated that there was virtually no wave transmission through the first breakwater located ½ mile from the embankment. Table 11 lists the incident and transmitted wave heights for the breakwater at the southeast corner of the reservoir and the resultant wave run-up (see Figure 6 for wind direction). The wave run-up was estimated to range from 4.2 to 9.5 ft for the 200 mph design condition and from 2.5 to 5.4 ft for the 105 mph with PMP design condition.

Evaluation of Wave Run-up and Internal Breakwaters

The effects of the circle with spokes breakwater on wave run-up are summarized in Table 12. All cases shown are for a 12 ft deep reservoir. For this breakwater configuration, wave heights at each of the breakwater structures are presented in addition to the wave run-up (see Figure 7 for wind directions). The northeast-southwest wind direction produced the highest values for wave run-up. For this wind direction, the wave run-up was estimated to range from 7.3 to 15.5 ft for the 200 mph design condition and from 4.7 to 10.1 ft for the 105 mph with PMP design condition.

Table 13 lists the freeboard requirements and embankment heights if internal breakwaters were installed and provides a comparison to the embankment heights for the cases without internal breakwaters. The embankment height is the sum of effective depth, i.e., depth plus rainfall, wind set-up and wave run-up. The peripheral wall could reduce the embankment height by at least 4 feet. The circle breakwater would not be as effective at reducing freeboard and may reduce the embankment height by only about 1 ft.

6. SUMMARY AND CONCLUSIONS

The design conditions selected by the District and USACE and evaluated in this Work Order included:

- 200 mph wind with no rainfall
- 105 mph wind with the PMP

Several other variables including fetch, depth, slope and type of surface on the embankment were modified to simulate a range of design conditions. The design conditions were evaluated with and without internal breakwaters that could reduce wave run-up on the embankments.

The wave growth and wave theory sections of the ACES model were used to identify the wave characteristics that could occur under the design conditions. The wave length, period and height all increase with increasing fetch, depth and wind speed. The relationship between wave height and fetch was developed and presented in Figure 5. The figure indicates that there is an initial sharp rise in wave height with increasing fetch. However, beyond a distance of about 2 to 4 miles, the curve becomes relatively flat indicating that an increase in fetch has a lesser effect on wave height.

Wind Set-up calculations were made for each of the cases evaluated. A minor adjust was made to the water depth in the calculations to account for the increased depth at the embankment. Wind set-up values were estimated to range 1 to 10 ft.

The wave-run-up module of the ACES model was used to estimate wave run-up for each of the 32 cases evaluated. Wave run-up increases with an increase in incident wave height and with slope of embankment. The wave run-up is not directly related to water depth and therefore is not related to wind set-up. Wave run-up is indirectly related to water depth because depth affects the incident wave height. On the flatter slope, the run-up was greater for a smooth surface; on the steeper slope, run-up was greater for the rough surface.

Two configurations of internal breakwaters were evaluated; a peripheral wall approximately ½ mile inside of the embankment, and a circle breakwater in the middle of the reservoir with several spokes radiating toward the embankments. For each of the configurations and wind directions evaluated wind and waves would encounter two breakwaters. Therefore, a step-wise approach was taken to develop the wave run-up on the reservoir embankment. The effects of wind set-up on the internal breakwaters was also considered in the analysis because if wind set-up is causing water to pile up at the internal breakwaters, wave transmission will be higher and the breakwaters will not be as effective at reducing wave run-up. As the design concepts for the internal breakwaters become more defined it would be useful to run a sensitivity analysis for the wind set-up assumption to determine the overall effect of this assumption on freeboard requirements.

The peripheral wall could reduce the embankment height by at least 4 7 feet. The circle breakwater would not be as effective at reducing freeboard and may reduce the embankment height by only about 1 ft.

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Evaluation of Wave Run-up and Internal Breakwaters

TABLES

Table 1. Wave Characteristics					
Wind Speed (mph)	**Depth (ft)**	**Fetch (miles)**	**Wave Length (ft)**	**Wave Period (sec)**	**Wave Height (ft)**
200	12	8.5	112	6.1	7.6
200	12	3.8	96	5.3	7.2
200	15	8.5	128	6.3	8.8
200	15	3.8	108	5.5	8.2
105	16.5	8.5	99	5.0	6.3
105	16.5	3.8	80	4.3	5.3
105	19.5	8.5	108	5.1	6.8
105	19.5	3.8	85	4.3	5.5

Table 2. Wind Set-up Calculations					
Wind Speed (mph)	**Fetch (miles)**	**Depth (ft)**	**Wind Set-up (ft)**	**Adjusted Depth (ft)**	**Adjusted Wind Set-up (ft)**
200	8.5	12	11.9	15.0	9.8
200	3.8	12	6.5	13.6	5.9
200	8.5	15	9.8	17.4	8.5
200	3.8	15	5.4	16.3	5.0
105	8.5	16.5	2.2	17.0	2.1
105	3.8	16.5	1.1	16.8	1.1
105	8.5	19.5	1.8	20.0	1.8
105	3.8	19.5	0.9	19.7	0.9

Evaluation of Wave Run-up and Internal Breakwaters

Table 3 Case 1: 200 mph; 12 ft depth; 3:1 slope, No Internal Structures									
Case 1	**Fetch Length (miles)**	**Dam Surface**	**Wave Height (ft)**	**Rainfall Depth (ft)**	**Effective Depth (ft)**	**Wave Run-up (ft)**	**Wind Setup (ft)**	**Minimum Freeboard (ft)**	**Elev. of Embankment (ft)**
A	8.5	Rough	7.6	0.0	12.0	7.3	9.8	17.1	29.1
B	3.8	Rough	7.2	0.0	12.0	6.5	5.9	12.4	24.4
C	8.5	Smooth	7.6	0.0	12.0	12.7	9.8	22.5	34.5
D	3.8	Smooth	7.2	0.0	12.0	10.8	5.9	16.7	28.7

Table 4 Case 2: 200 mph; 12 ft depth; 0.5:1 slope, No Internal Structures									
Case 2	**Fetch Length (miles)**	**Dam Surface**	**Wave Height (ft)**	**Rainfall Depth (ft)**	**Effective Depth (ft)**	**Wave Run-up (ft)**	**Wind Setup (ft)**	**Minimum Freeboard (ft)**	**Elev. of Embankment (ft)**
A	8.5	Rough	7.6	0.0	12.0	14.6	9.8	24.4	36.4
B	3.8	Rough	7.2	0.0	12.0	13.5	5.9	19.4	31.4
C	8.5	Smooth	7.6	0.0	12.0	11.8	9.8	21.6	33.6
D	3.8	Smooth	7.2	0.0	12.0	10.8	5.9	16.7	28.7

Table 5 Case 3: 200 mph; 15 ft depth; 3:1 slope, No Internal Structures									
Case 3	**Fetch Length (miles)**	**Dam Surface**	**Wave Height (ft)**	**Rainfall Depth (ft)**	**Effective Depth (ft)**	**Wave Run-up (ft)**	**Wind Setup (ft)**	**Minimum Freeboard (ft)**	**Elev. of Embankment (ft)**
A	8.5	Rough	8.8	0.0	15.0	8.3	8.5	16.8	31.8
B	3.8	Rough	8.2	0.0	15.0	7.2	5.0	12.2	27.2
C	8.5	Smooth	8.8	0.0	15.0	14.2	8.5	22.7	37.7
D	3.8	Smooth	8.2	0.0	15.0	11.8	5.0	16.8	31.8

Evaluation of Wave Run-up and Internal Breakwaters

Table 6
Case 4: 200 mph; 15 ft depth; 0.5:1 slope, No Internal Structures

Case 4	Fetch Length (miles)	Dam Surface	Wave Height (ft)	Rainfall Depth (ft)	Effective Depth (ft)	Wave Run-up (ft)	Wind Setup (ft)	Minimum Freeboard (ft)	Elev. of Embankment (ft)
A	8.5	Rough	8.8	0.0	15.0	12.7	8.5	21.2	36.2
B	3.8	Rough	8.2	0.0	15.0	10.2	5.0	15.2	30.2
C	8.5	Smooth	8.8	0.0	15.0	9.4	8.5	17.9	32.9
D	3.8	Smooth	8.2	0.0	15.0	7.6	5.0	12.6	27.6

Table 7
Case 5: 105 mph W/ PMP; 12 ft depth; 3:1 slope, No Internal Structures

Case 5	Fetch Length (miles)	Dam Surface	Wave Height (ft)	Rainfall Depth (ft)	Effective Depth (ft)	Wave Run-up (ft)	Wind Setup (ft)	Minimum Freeboard (ft)	Elev. of Embankment (ft)
A	8.5	Rough	6.3	4.5	16.5	5.6	2.2	12.3	24.3
B	3.8	Rough	5.3	4.5	16.5	4.5	1.1	10.1	22.1
C	8.5	Smooth	6.3	4.5	16.5	9.4	2.2	16.1	28.1
D	3.8	Smooth	5.3	4.5	16.5	7.4	1.1	13.0	25.0

Table 8
Case 6: 105 mph W/ PMP; 12 ft depth; 0.5:1 slope, No Internal Structures

Case 6	Fetch Length (miles)	Dam Surface	Wave Height (ft)	Rainfall Depth (ft)	Effective Depth (ft)	Wave Run-up (ft)	Wind Setup (ft)	Minimum Freeboard (ft)	Elev. of Embankment (ft)
A	8.5	Rough	6.3	4.5	16.5	11.8	2.2	18.5	30.5
B	3.8	Rough	5.3	4.5	16.5	9.7	1.1	15.3	27.3
C	8.5	Smooth	6.3	4.5	16.5	8.8	2.2	15.5	27.5
D	3.8	Smooth	5.3	4.5	16.5	7.2	1.1	12.8	24.8

Evaluation of Wave Run-up and Internal Breakwaters

Table 9
Case 7: 105 mph W/ PMP; 15 ft depth; 3:1 slope, No Internal Structures

Case 7	Fetch Length (ft)	Dam Surface	Wave Height (ft)	Rainfall Depth (ft)	Effective Depth (ft)	Wave Run-up (ft)	Wind Setup (ft)	Minimum Freeboard (ft)	Elev. of Embankment (ft)
A	8.5	Rough	6.8	4.5	19.5	6.0	1.8	12.3	27.3
B	3.8	Rough	5.5	4.5	19.5	4.7	0.9	10.1	25.1
C	8.5	Smooth	6.8	4.5	19.5	10.0	1.8	16.3	31.3
D	3.8	Smooth	5.5	4.5	19.5	7.7	0.9	13.1	28.1

Table 10
Case 8: 105 mph W/ PMP; 15 ft depth; 0.5:1 slope, No Internal Structures

Case 8	Fetch Length (ft)	Dam Surface	Wave Height (ft)	Rainfall Depth (ft)	Effective Depth (ft)	Wave Run-up (ft)	Wind Setup (ft)	Minimum Freeboard (ft)	Elev. of Embankment (ft)
A	8.5	Rough	6.8	4.5	19.5	12.7	1.8	19.0	34.0
B	3.8	Rough	5.5	4.5	19.5	10.2	0.9	15.6	30.6
C	8.5	Smooth	6.8	4.5	19.5	9.4	1.8	15.7	30.7
D	3.8	Smooth	5.5	4.5	19.5	7.6	0.9	13.0	28.0

Evaluation of Wave Run-up and Internal Breakwaters

Table 11. Effects of Peripheral Wall on Wave Run-up

Embankment (slope, roughness)	Incident Wave Height H(i) (ft)	Transmitted Wave Height H(t) (ft)	Wave Run-up (ft)
200 mph Wind			
3:1, rough	7.5	2.4	4.2
3:1, smooth	7.5	2.4	6.7
0.5:1, rough	7.5	2.4	9.5
0.5:1, smooth	7.5	2.4	7.4
105 mph Wind, PMP			
3:1, rough	6.0	1.5	2.5
3:1, smooth	6.0	1.5	3.9
0.5:1, rough	6.0	1.5	5.4
0.5:1, smooth	6.0	1.5	4.1

Table 12. Effects of Circle Structure on Wave Run-up

Embankment (slope, roughness)	1st Breakwater		2nd Breakwater		Wave Run-up (ft)
	H(i) (ft)	H(t) (ft)	H(i) (ft)	H(t) (ft)	
200 mph Wind, North-South Direction					
3:1, rough	6.9	0.7	7.5	1.7	4.9
3:1, smooth	6.9	0.7	7.5	1.7	7.7
0.5:1, rough	6.9	0.7	7.5	1.7	10.8
0.5:1, smooth	6.9	0.7	7.5	1.7	9.0
200 mph wind, Northwest-Southeast Direction					
3:1, rough	6.9	0.8	7.1	1.1	7.3
3:1, smooth	6.9	0.8	7.1	1.1	12.1
0.5:1, rough	6.9	0.8	7.1	1.1	15.5
0.5:1, smooth	6.9	0.8	7.1	1.1	12.7
105 mph wind, PMP, North-South Direction					
3:1, rough	4.8	0.5	5.0	0.8	2.8
3:1, smooth	4.8	0.5	5.0	0.8	4.4
0.5:1, rough	4.8	0.5	5.0	0.8	6.1
0.5:1, smooth	4.8	0.5	5.0	0.8	4.6
105 mph wind, PMP, Northwest-Southeast Direction					
3:1, rough	4.8	0.5	4.4	0.4	4.7
3:1, smooth	4.8	0.5	4.4	0.4	7.7
0.5:1, rough	4.8	0.5	4.4	0.4	10.1
0.5:1, smooth	4.8	0.5	4.4	0.4	7.7

Evaluation of Wave Run-up and Internal Breakwaters

Table 13. Comparison of Freeboard Requirements							
Case	**Effective Depth (ft)**	**Wind Set-up (ft)**	**Wave Run-up (ft)**	**Freeboard With Breakwaters (ft)**	**Embankment Height with Breakwaters (ft)**	**Embankment Height without Breakwaters (ft)**	**Reduction in Embankment Height (ft)**
Peripheral Wall							
1 A	12	6.0	4.2	10.2	22.2	29.1	6.9
1 C	12	6.0	6.7	12.7	24.7	34.5	9.8
2 A	12	6.0	9.5	15.5	27.5	36.4	8.9
2 C	12	6.0	7.4	13.4	25.4	33.6	8.2
5 A	16.5	1.2	2.5	8.2	20.2	24.3	4.1
5 C	16.5	1.2	3.9	9.6	21.6	28.1	6.5
6 A	16.5	1.2	5.4	11.1	23.1	30.5	7.4
6 C	16.5	1.2	4.1	9.8	21.8	27.5	5.7
Circle Structure							
1 A	12	8.0	7.3	15.3	27.3	29.1	1.8
1 C	12	8.0	12.1	20.1	32.1	34.5	2.4
2 A	12	8.0	15.5	23.5	35.5	36.4	0.9
2 C	12	8.0	12.7	20.7	32.7	33.6	0.9
5 A	16.5	1.6	4.7	10.8	22.8	24.3	1.5
5 C	16.5	1.6	7.7	13.8	25.8	28.1	2.3
6 A	16.5	1.6	10.1	16.2	28.2	30.5	2.3
6 C	16.5	1.6	7.7	13.8	25.8	27.5	1.7

Cases:

1:	200 mph wind, 12 ft depth, 3:1 slope	A – rough, C – smooth
2:	200 mph wind, 12 ft depth, 0.5: 1 slope	A – rough, C – smooth
5:	105 mph wind, PMP, 12 ft depth, 3:1 slope	A – rough, C – smooth
6:	105 mph wind, PMP, 12 ft depth, 0.5:1 slope	A – rough, C - smooth

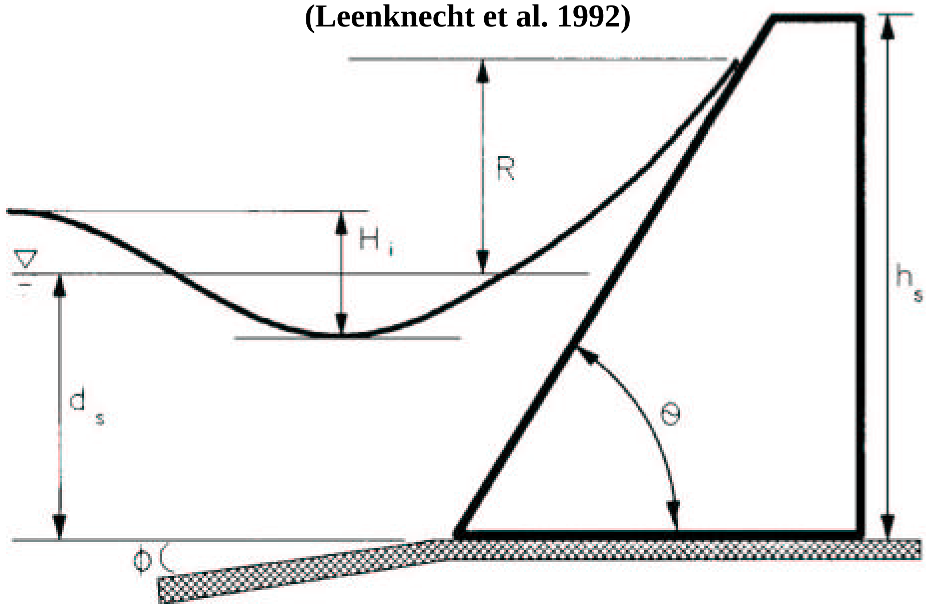
Evaluation of Wave Run-up and Internal Breakwaters

FIGURES

Figure 1: Reservoir Embankment and Fetches Modeled

Evaluation of Wave Run-up and Internal Breakwaters

Figure 2: Definition of Wave Run-Up Parameters
(Leenknecht et al. 1992)



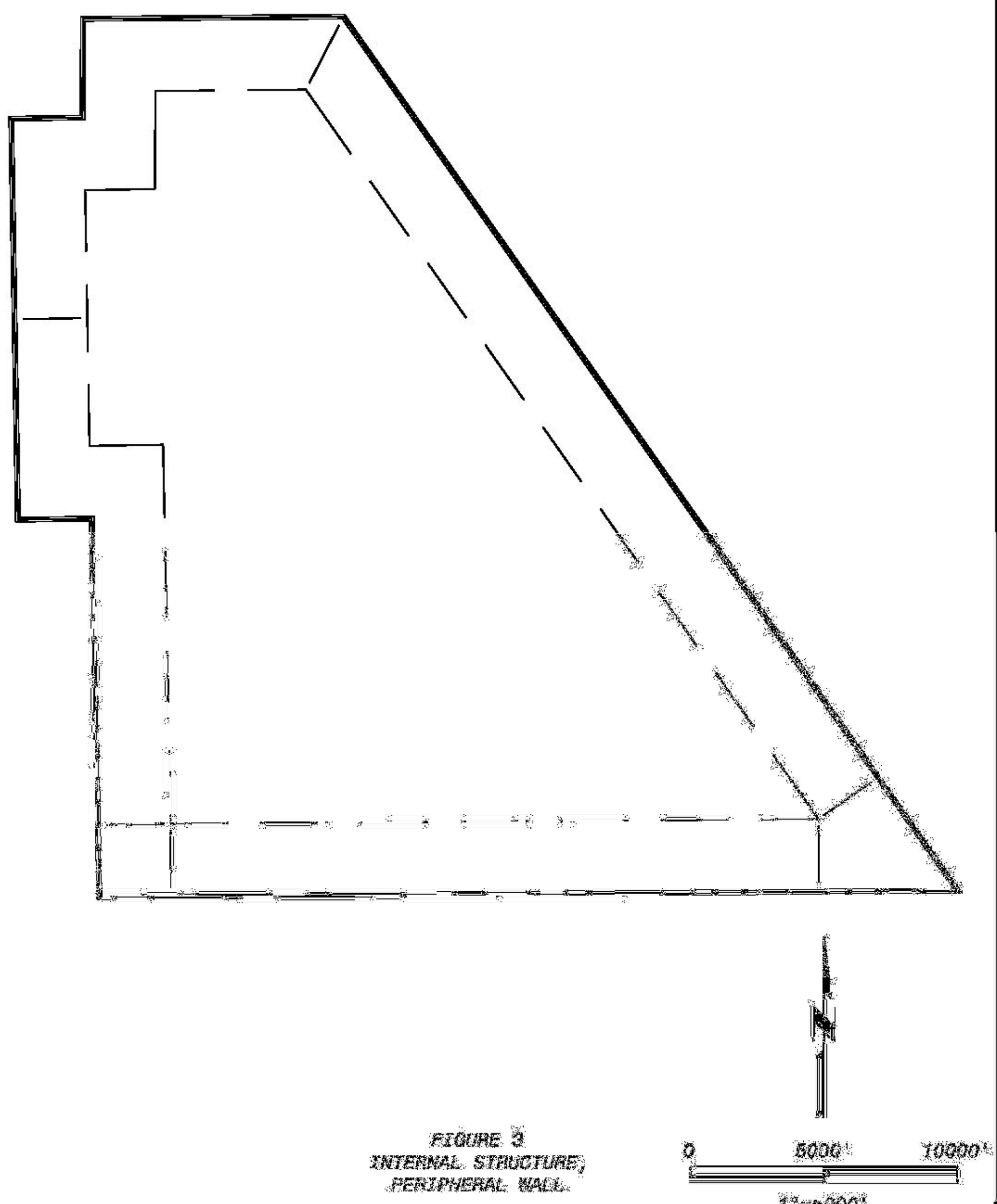


FIGURE 3
INTERNAL STRUCTURE,
PERIPHERAL WALL

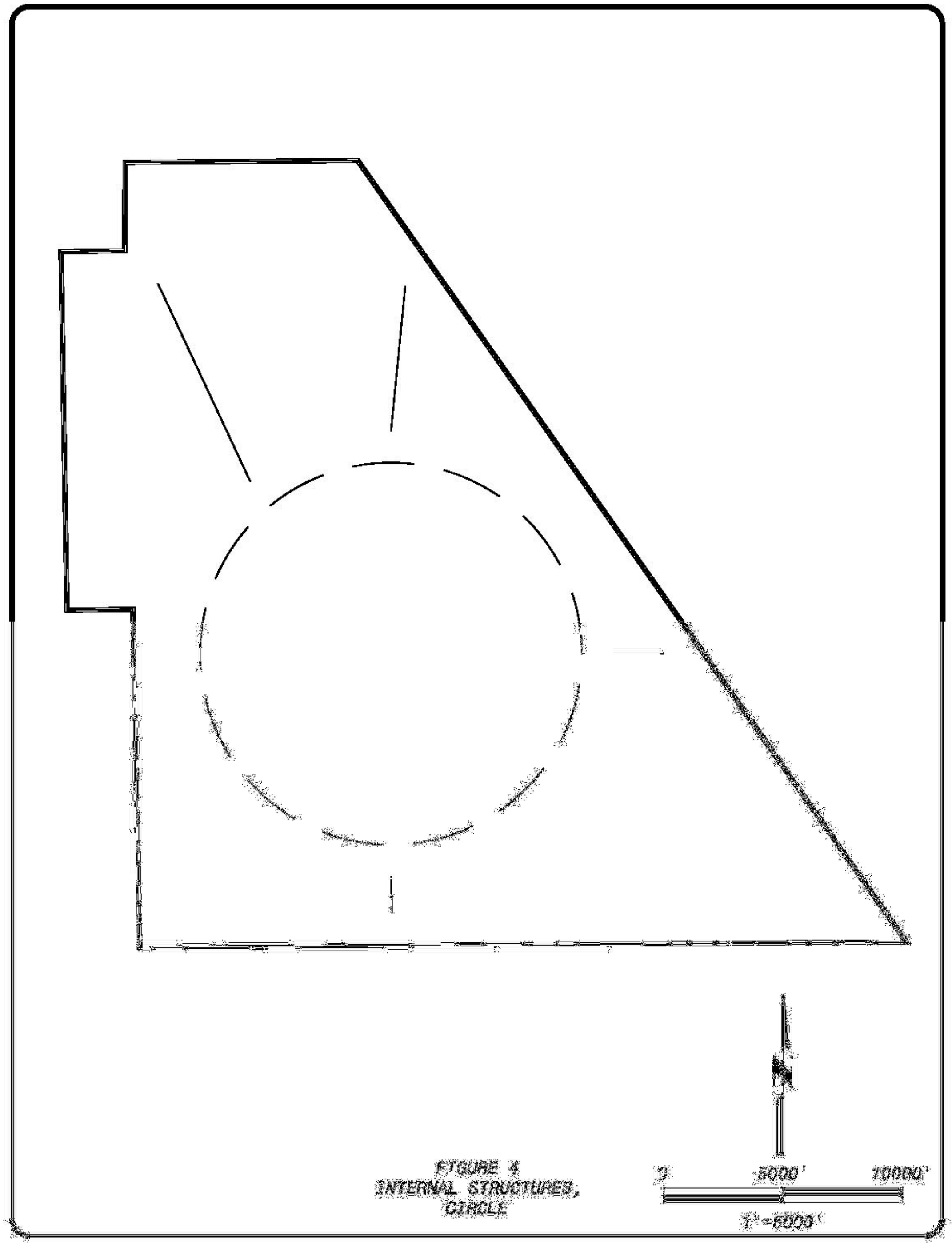
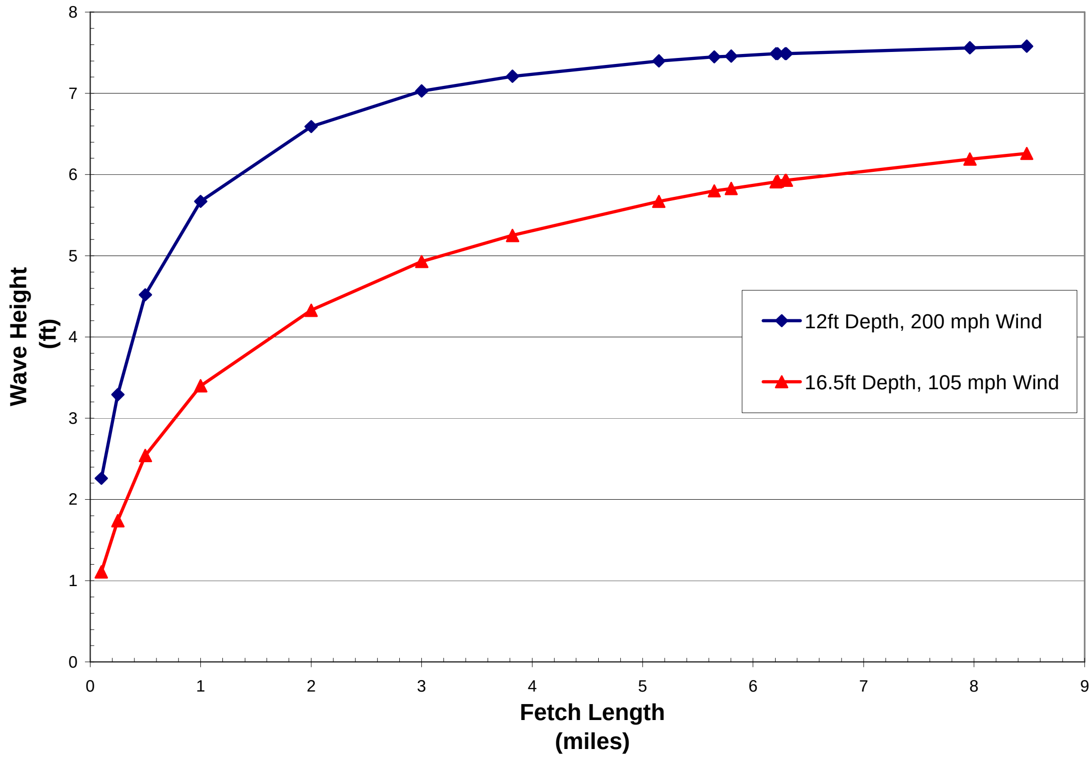


FIGURE 4
INTERNAL STRUCTURES,
CIRCLE

Evaluation of Wave Run-up and Internal Breakwaters

Figure 5: Wave Height versus Fetch Length



Evaluation of Wave Run-up and Internal Breakwaters

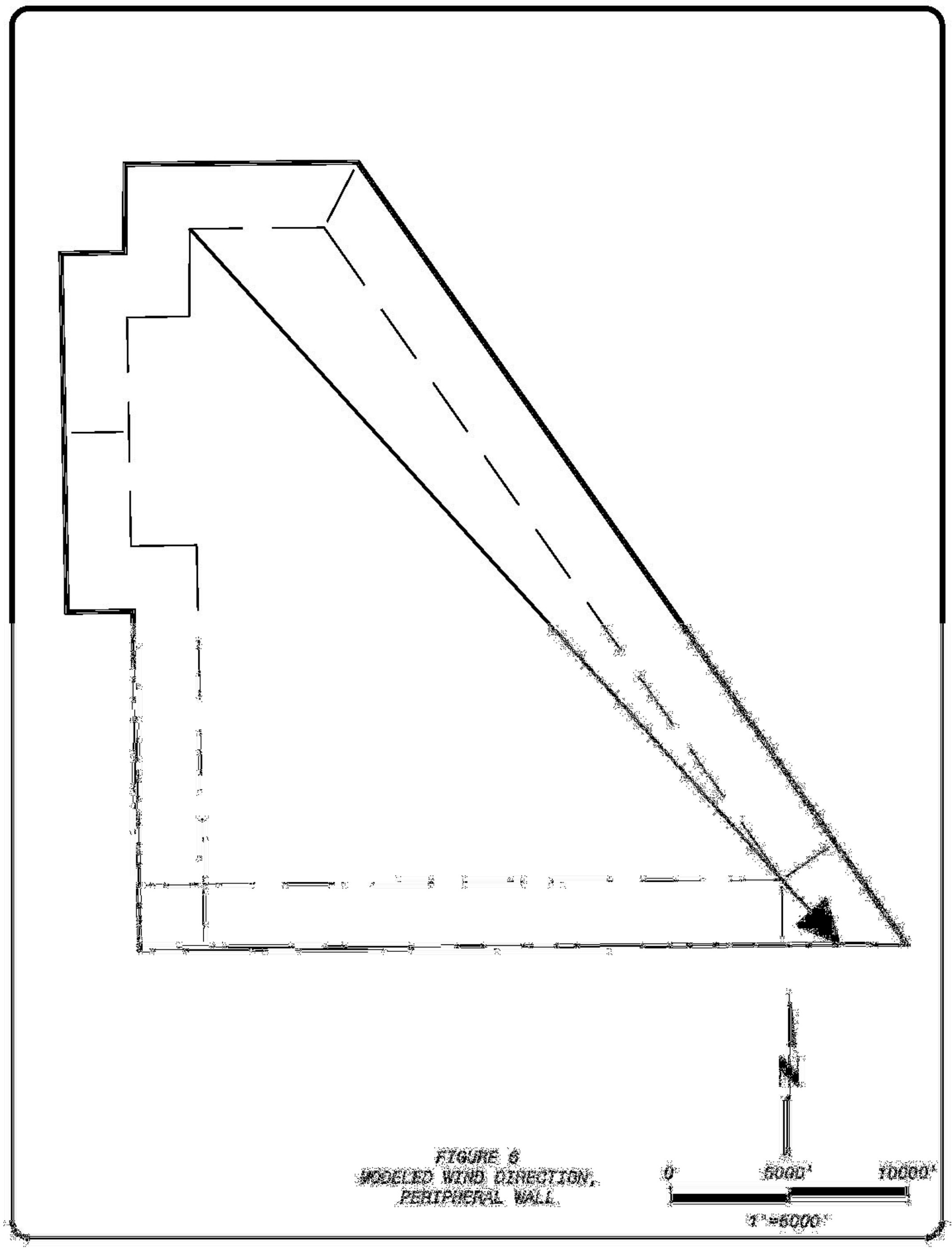


FIGURE 6
MODELED WIND DIRECTION,
PERIPHERAL WALL

Evaluation of Wave Run-up and Internal Breakwaters

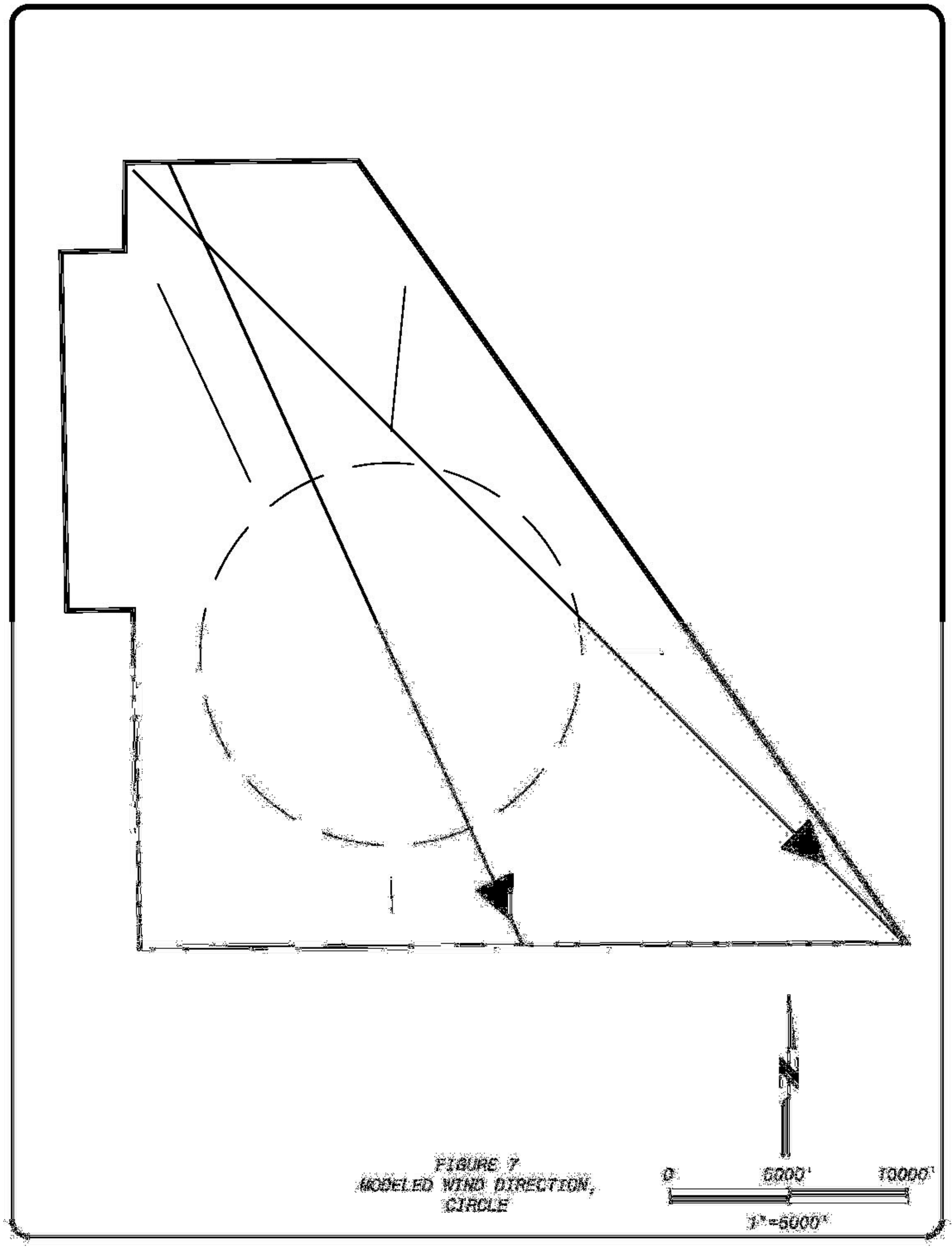


FIGURE 7
MODELED WIND DIRECTION,
CIRCLE